# CHAPTER 7

# SPECIAL CONSIDERATIONS FOR FLOOD WALLS

## Section I. General Characteristics

7-1. Introduction. The principal function of a flood wall is to prevent flooding (inundation) of adjacent land. A flood wall is subject to hydraulic loading on one side which is resisted by little or no earth loading on the other side. The two principal types of flood walls are inland and coastal. Inland flood walls typically are installed along a riverbank and are subjected to design loadings (pool to freeboard line) for periods of hours or days (long-term loadings). Coastal flood walls are primarily subjected to short-term loadings (waves from hurricanes along with wind/tide high water surges). The wave loadings are dynamic in nature and act upon the structure for only a few seconds each. Concurrent high winds can prevent any emergency maintenance during a storm. Utility line crossings through a flood wall require careful attention to allow for independent movement of the utility lines and the wall, which requires special expansion joint details.

7-2. Rationale for Loading Cases.

a. Design Water Level.

(1) The hydraulic data required for determining the design water level should be listed in the hydrologic/hydraulic appendix of the pertinent planning document for the project. The flow characteristics noted in historical records and indicated from detailed observation of existing conditions will usually be basic to the design of inland flood walls. Coastal flood walls will frequently require hurricane surge simulation studies and wave setup estimates. Wave overtopping can cause severe scour at or near the protected side of the stem. See paragraph 3-24 for information on surge and wave loads.

(2) Factors that influence the water surface profile and level of protection, and that can reasonably be quantified, are included in the design water level; not the freeboard. Some examples of these factors are:

(a) Changed conveyance, due to changing bed form, sedimentation or scour, and vegetation growth or removal.

(b) Dynamic surges, and super elevation.

(c) Ice, debris, and local anomalies.

(d) Transverse slope due to water flowing out of or into the channel or differences in velocity head between the channel and overbank locations.

(e) Profile instabilities associated with braids, meanders, etc.

(f) Energy losses due to changing flow area, e.g. constrictions (bridges), abrupt expansions, and bends.

(g) Future changes in flood flows due to changes in the watershed.

b. Freeboard. The freeboard is the marginal height of wall provided above the design water level. Freeboard is designed to accomplish design objectives and allow for uncertainty in a water surface profile.

(1) Examples of design objectives are:

(a) Assurance of initial overtopping at the most desirable (least hazardous) location.

(b) Reduced volume of wave overtopping.

(c) Extension of interval between major maintenance such as removal of sediment deposition.

(2) Freeboard allowances for water surface uncertainty are allowances that are not otherwise specifically accounted for because they are considered too small to require specific determination or because they are too intractable to be quantified. Those factors that influence the water surface profile, and level of protection, and that can reasonably be quantified are included in the design water level; not the freeboard.

(3) Wall settlement is identified as a separate increment added to the wall height for that purpose and is not included in the freeboard.

(4) Freeboard design should be refined as a study progresses and not left entirely to a detailed design phase. The amount of effort and corresponding refinement for a given phase is a function of the importance and cost of freeboard relative to the overall plan. For an early reconnaissance phase it will generally be satisfactory to use quickly estimated freeboard values of generally accepted default values. Default values of 2 feet on agricultural and 3 feet on urban flood walls have been generally accepted. As the study progresses, these early estimated or default values will be replaced by values arrived at by a design process.

(5) When large non-breaking waves are normal incident to the stem of the flood wall, the amount of freeboard will be determined by the amount of overtopping allowed. It is important to remember that such overtopping can cause significant scour on the protected (toe) side of the wall. This potential for scour can require rigid paving within a 20- to 30-foot area of the wall.

c. Loading Cases. For determining water and soil loads acting on flood walls, refer to Chapter 3. Section I of Chapter 4 discusses loading cases.

## Section II. Seepage Control

7-3. General Considerations. Water-retaining structures are subject to through-seepage, underseepage, and seepage around their sides or ends. Seepage control is a primary consideration of flood wall design. Uncontrolled seepage may result in water pressures and uplift forces on the wall base in excess of design assumptions and consequent structural instability. Excessive porewater pressures in foundation materials near the landside toe of a wall may create "quick" conditions evidenced by sand boils or heaving. Emerging seepage may have sufficient velocity to move cohesionless foundation materials and erode the wall foundation (piping). Seepage control entails the design of measures to ensure that seepage pressures and velocities are maintained below tolerable values. Properly controlled seepage, even if quantities are large, presents no hazard. Since flood walls are often built in congested areas, it is often necessary to pump seepage out of the protected area. While the seepage quantity is often small compared to other sources, it is occasionally appropriate to consider seepage control measures for the purpose of reducing seepage quantities. Inadequate seepage control, as shown by one example in Figure 7-1, may jeopardize the stability of a flood wall. In flood walls, control of through-seepage is provided for by water stops (paragraph 7-13). Seepage around the wall is controlled by specially designed and constructed levee wrap-around sections (paragraph 7-12). Flood walls are usually provided with a toe drain to control local underseepage along the flood wall base, as shown in Figure 7-2. As flood walls are usually founded on alluvial materials, pervious zones of significant thickness are often present at some depth below relatively impervious top stratum materials and may be hydraulically connected to the river. Because of the horizontal stratification of alluvial deposits, the horizontal permeability may be greatly in excess of the vertical permeability. The combination of these conditions may allow seepage to be readily conducted landward beneath the flood wall. Where flood walls are underlain by such pervious strata (the usual case), analysis may indicate the need for underseepage controls in addition to the toe drain. Underseepage control measures vary because the selection and design of an appropriate control scheme is highly dependent on site-specific conditions, particularly the stratification and permeability of foundation materials, availability of right-of-way, and local construction practices and costs. Various types of underseepage control measures are discussed in paragraph 7-4.

7-4. Underseepage Control. The focus of underseepage analysis is to calculate the expected exit gradient at the landside toe of a levee or flood wall and compare its value to a theoretically critical value, the critical gradient (typically 0.8 to 1.0). To provide some conservatism, underseepage controls are provided where the calculated gradient exceeds an allowable gradient, typically 0.5 to 0.8. For calculating the exit gradient, assessing the need for underseepage controls, and designing such controls, the foundation conditions are normally assumed to be a two-layer system consisting of a relatively impervious top stratum overlying a pervious substratum. Detailed analysis

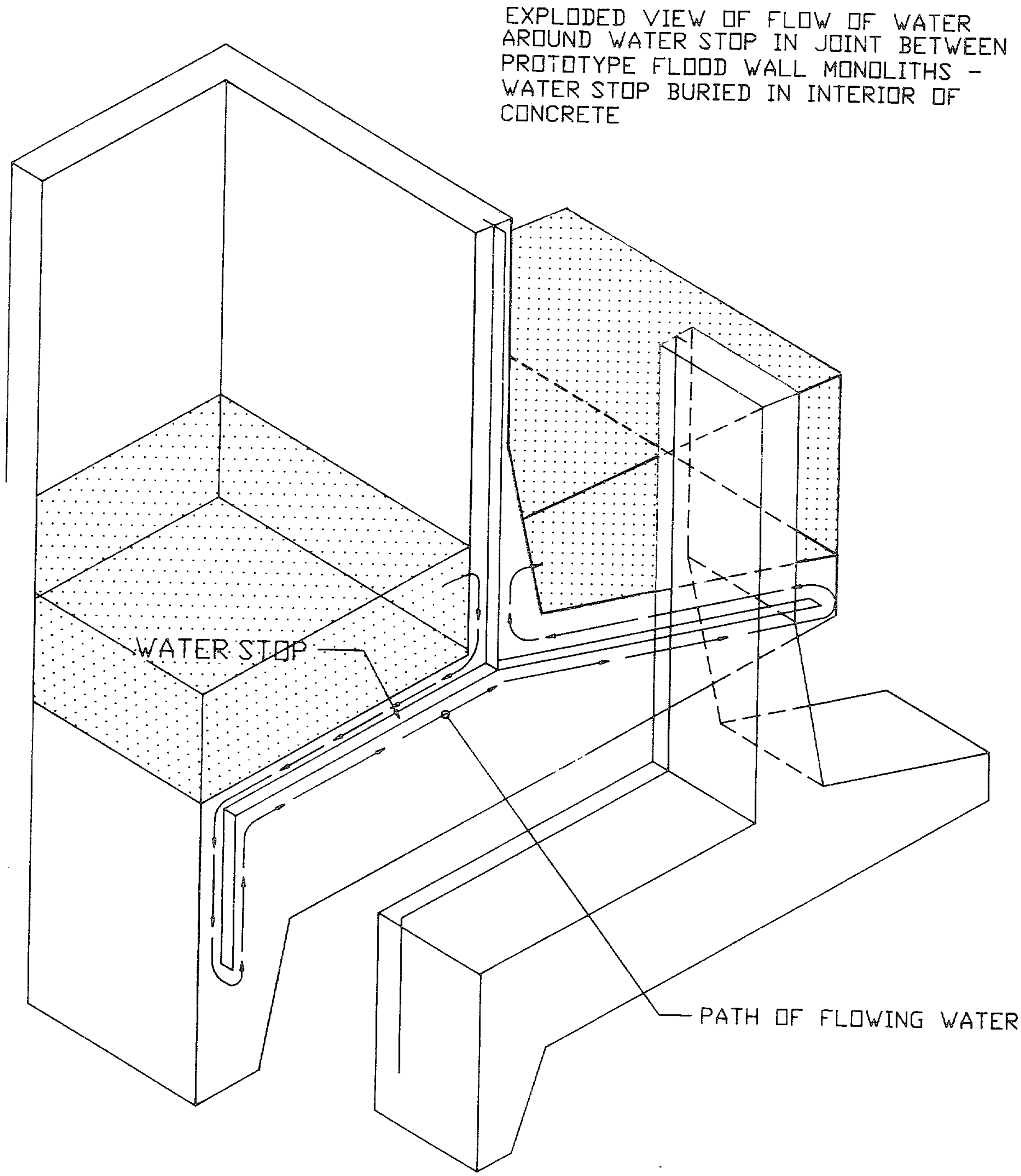
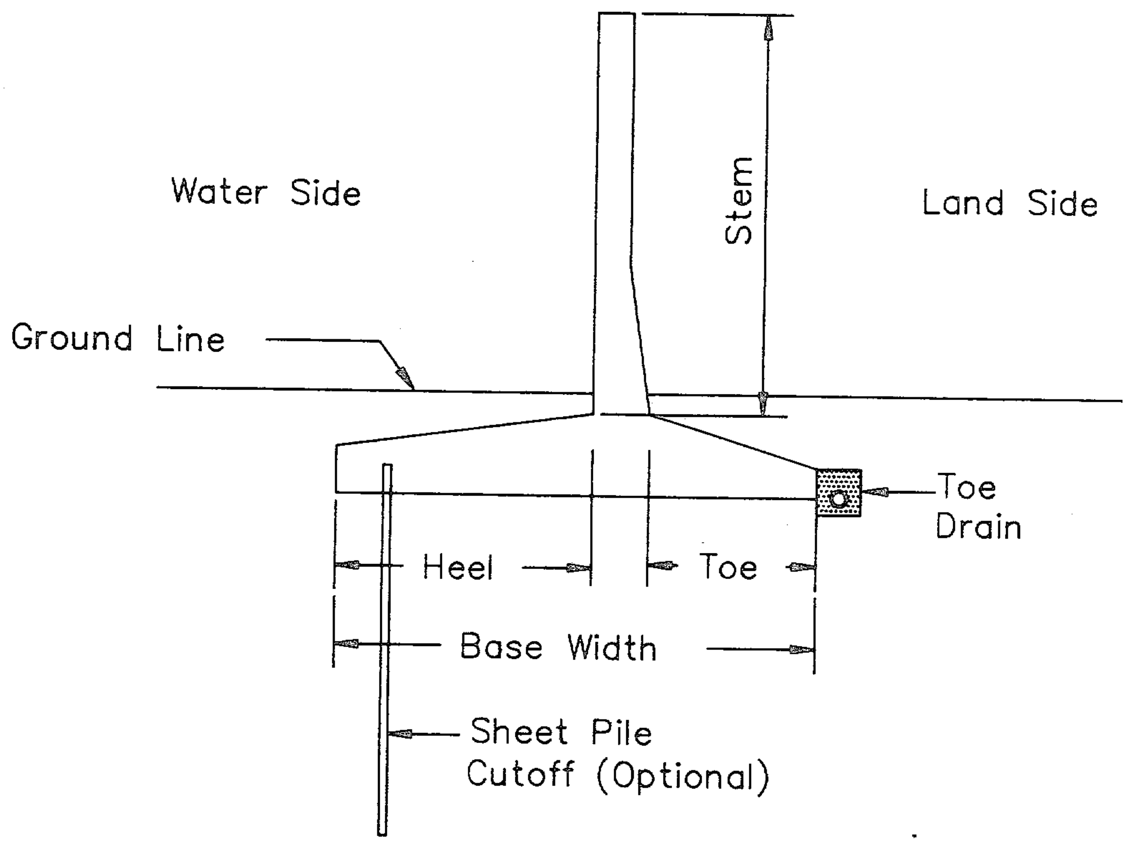



Figure 7-1.  Flow around interior embedded water stop in the base



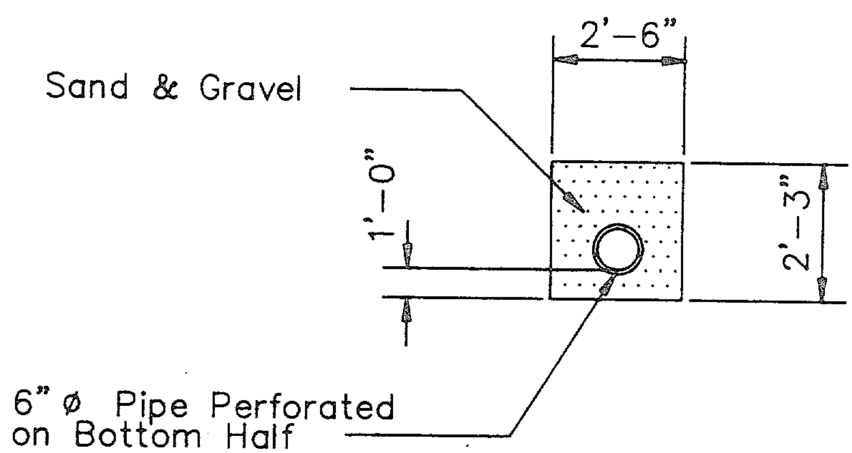
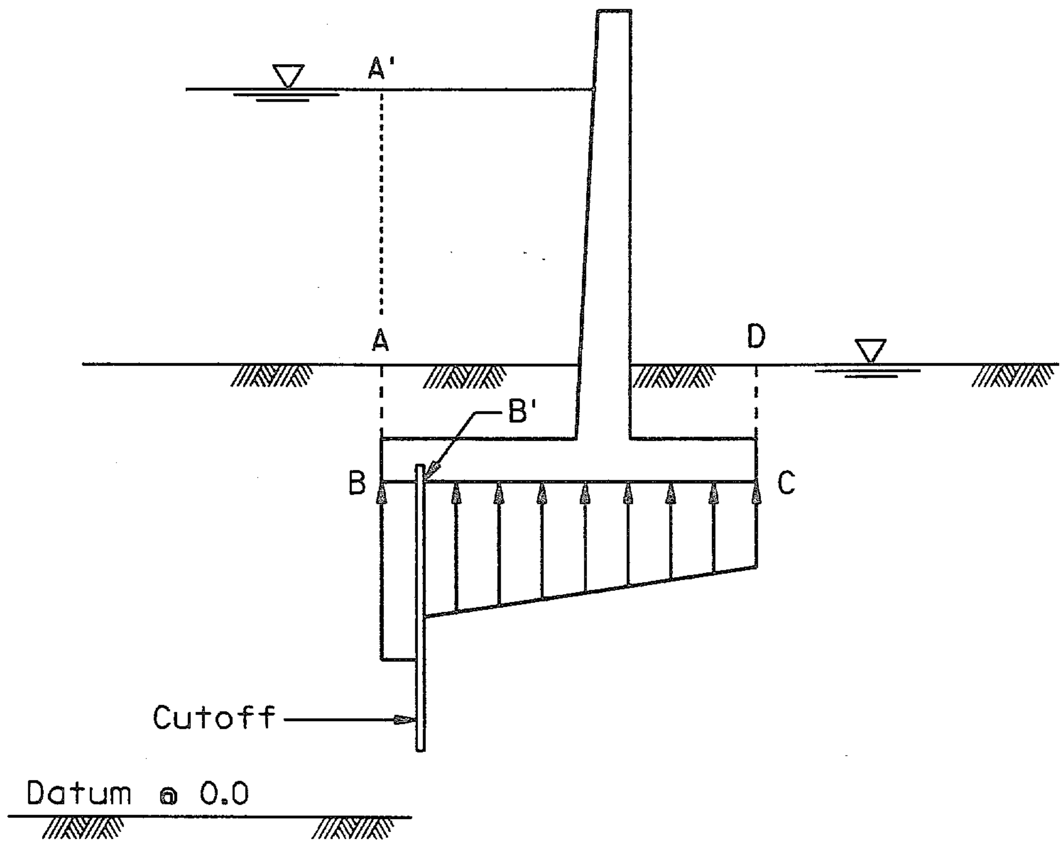




Figure 7-2. T-type flood wall--horizontal base

procedures are contained in EM 1110-2-1913 and WES Technical Memorandum 3-424 (US Army Engineer Waterways Experiment Station, 1956). In some instances, where complex problems of geometry, anisotropy, and foundation layering exist, flow nets and/or finite element seepage analyses may be necessary. Types of underseepage control measures are described in the following paragraphs. Additional discussion is given in paragraphs 3-23 and 6-6.

a. Cutoffs. A cutoff penetrating the pervious stratum beneath the wall is the most positive means of controlling seepage. A cutoff can consist of an excavated trench backfilled with impervious compacted earth, a slurry trench, an extension of a concrete shear key, or a sheet pile wall. A cutoff is usually located at the end of the wall footing on the unprotected (heel) side. A cutoff must penetrate approximately 95 percent or more of the pervious strata before significant reductions in the quantity of flow can be realized; however, partial cutoffs can be somewhat effective in reducing uplift pressures on the wall base. Deep cutoffs will often interfere with the normal exchange of groundwater between an aquifer and a river during non-flood periods and should only be considered where detailed hydrogeologic studies have been made in this regard. The decision as to the type and depth of a cutoff should be based on an underseepage analysis considering actual site conditions. A steel sheet pile cutoff is not entirely watertight due to leakage at the interlocks but can significantly reduce the possibility of piping of coarse-grained material in the foundation. The effectiveness of a properly interlocked steel sheet pile cutoff through a coarse-grained stratum in reducing uplift can be assumed to be up to 50 percent. The design uplift diagram, as shown in Figure 7-3, should be drawn with a pressure head at point B on the unprotected side of the the cutoff equal to the full head of water on the unprotected side (neglecting any reduction in pressure due to head loss from seepage effects). The pressure head on the protected side of the cutoff at point B should equal the pressure at point B reduced by up to 50 percent of the difference between the full head value on the unprotected side and the pressure head at the end of the toe of the wall. The pressure head at the toe of the wall can be computed based on the seepage path from the cutoff wall to the saturated level on the protected side. If the effectiveness of the steel sheet pile cutoff is assumed to be greater than 50 percent, it should be based on actual experience of similar conditions and justified accordingly. An example of a flood wall with a sheet pile cutoff is shown in example 5 of Appendix N. A sheet pile cutoff is less effective in fine-grained material than in coarse-grained material because cohesion may allow cracking and separation of the soil away from the sheet pile. Bearing value of steel sheet piling should be neglected.

b. Toe Drains. All inland flood walls should be provided with a landside toe drain similar to that shown in Figure 7-2. Coastal flood walls should be analyzed to determine if such drains are needed. The toe drain, which runs parallel to the wall at the landside edge of the footing, provides a positive outlet for local underseepage and a check for controlling piping and/or excessive uplift pressure beneath the base slab. For walls on impervious foundations, the toe drain may be adequate to control all underseepage; for walls on pervious foundations, additional seepage control measures will




Total head at B = $El_{A'}$

Pressure head at B = $El_{A'} - El_B$

Pressure head at B' =
$(El_{A'} - El_B) - 0.5$ [(Pressure head at B) - (Pressure head at C)]

Total head at C = $El_D + \left[\frac{DC}{DCB'}\right](El_{A'} - El_D)$

Pressure head at C = Total head at C - $El_C$

Figure 7-3. Uplift pressures for a wall with a sheet pile cutoff usually be required. In the case of pile-founded walls, the toe drain should be adequate to protect against "roofing," the loss of material from beneath the wall base. The drain should never be located under the wall footing, in order to allow maintenance access and to avoid crushing the drain. A typical toe drain design will consist of a 6- to 8-inch-diameter pipe perforated on the bottom half and surrounded in all directions with 6 to 10 inches of filter material designed by the filter criteria in paragraph 6-6e. The collected water is usually disposed of by gravity outlets into ditches, ponding areas, or pump stations. The toe drain system should provide access for inspection and maintenance at changes in alignment and at intervals not to exceed 500 feet. Discharge pipes should be provided with check valves that will prevent the entrance of surface water.

c. Trench Drains. Where the impervious top stratum is thin or non-existent, a trench drain may be used to control underseepage in the vicinity of the flood wall toe. A trench drain is an enlarged variation of a toe drain. It extends from the ground surface through shallow pervious layers or into a pervious layer underlying a shallow surface blanket. The practical depth for construction of a trench drain depends on available excavation equipment and site dewatering requirements. The excavation, pipe placement, and backfilling of the trench should always be performed in the dry. To assure adequate capacity, the collector pipe should be sized considerably larger than computations indicate to be necessary. Backfill in a trench drain should conform to the filter criteria in paragraph 6-6e. A trench drain should be provided with inspection and maintenance access and backflow protection as described for toe drains. The seepage calculations for the quantity of flow should assume the tailwater elevation equal to that of the discharge of the trench drain. However, if water can pond on the landside of the wall, the calculations for uplift pressure should check whether a more critical uplift condition can occur for the ponded case.

d. Relief Wells. Pressure relief wells are used to reduce uplift pressures at depths in pervious layers which might otherwise cause sand boils and piping of foundation materials. Wells function to some extent as a controlled sand boil, relieving pressure by discharging water, but retaining materials with a screen and filter. Wells are advantageous where pervious strata are relatively thick or relatively deep. They are particularly useful in controlling large quantities of seepage in strata of pervious material having direct connections with the river. Another advantage of relief wells is the ease with which they can be constructed if piezometric pressures measured during high water indicate the need for additional underseepage control. Design of relief well systems is described in EM 1110-2-1905, EM 1110-2-1901, and WES Technical Memorandum 3-424 (US Army Engineer Waterways Experiment Station, 1956). The design entails selecting a spacing, size, and penetration for a line of wells that will result in acceptable gradients at points midway between the line of wells and at the flood wall toe. Relief wells are usually not very effective in intercepting near-surface seepage, and it is often wise to use them in combination with a toe drain. Relief wells should be pump-tested when installed. Because the efficiency of relief wells may deteriorate with time due to corrosion or bacterial incrustation, considerable monitoring and maintenance may be required to assure that the relief well system performs acceptably for the project life. To assess possible well deterioration, a representative number of wells should be periodically pump-tested, and the specific capacity (flow/drawdown) should be compared to the initial pump test results. To calculate uplift pressures on the wall, the potential head at the well line should be assumed equal to the average head in the plane of wells, a value obtained as part of the well design procedure in the cited references.

e. Riverside Impervious Blankets. Impervious riverside blankets (natural or constructed) overlying a pervious foundation are effective in reducing the quantity of seepage and to some extent are effective in reducing uplift pressures and gradients landside of the flood wall. Their effects may be analyzed using seepage analysis methods found in EM 1110-2-1913 and WES Technical Memorandum 3-424 (US Army Engineer Waterways Experiment Station, 1956). Riverside blankets may be constructed over thin natural impervious blankets to improve the effects of the natural blankets or they may be constructed directly on pervious material. Excessively steep riverbanks may make blanket construction impractical. Also, it is seldom feasible to construct blankets over exposed portions of the pervious layer under water. A noncontinuous blanket has serious drawbacks, as only a small area of pervious stratum left exposed may significantly reduce the blanket's effectiveness. Riverside impervious blankets need to overlap the riverside base of the flood wall to minimize the potential for rupture of the blanket by landward deflection of the flood wall when loaded. Riverside impervious blankets may be subject to scour at high river stages when they would be most needed, or may crack open if not continuously wet. To prevent such action, blankets should be protected immediately after construction. A well-designed and well-planted vegetative cover is ordinarily sufficient along straight reaches. Along outside curves of the river, the blankets should be protected with riprap or other positive protection.

f. Landside Seepage Berms. Landside seepage berms function by providing an increased landside top blanket thickness, thereby reducing the gradient. The berm also extends the seepage path by forcing the seepage exit landward. Seepage berms are typically 100 to 300 feet wide. As flood walls are usually built in areas where right-of-way cost or availability is insufficient for levee construction, seepage berms are rarely used in conjunction with flood walls. Procedures for seepage berm design are presented in WES Technical Memorandum 3-424 (US Army Engineer Waterways Experiment Station, 1956) and EM 1110-2-1913.

g. Grouting of Open Rock Joints. In cases where rock is shallow enough that flood walls can be founded directly on the rock, close examination of the rock surface is necessary to determine if open joints are present. Such joints can be detrimental to underseepage control and should be cleaned out and filled with grout before the concrete base is placed. If the possibility exists for seepage flow through porous or cavernous rock in the foundation, consideration should be given to installing a grout curtain.

## Section III. Foundation Considerations

7-5. Base Types. The T-wall is the most widely used flood wall type. T-walls are normally constructed with horizontal or sloped bases. The advantages of each type of base are as follows:

a. Horizontal Base (Figure 7-2).

(1) The volume of foundation excavation is usually less for a horizontal base and it is simpler to construct.

(2) Bearing values and base pressures for the two base types are not directly comparable. However, for equal heights, base pressures of the horizontal base generally are smaller because of its reduced earth load and slightly wider base.

b. Sloped Base (Figure 7-4).

(1) A sloped base may allow shortening or complete elimination of a key, thereby reducing excavation difficulties. Also, a shorter key will generate less moment in the heel adjacent to the key and will generally allow for a shorter base width to maintain overturning equilibrium.

(2) The deep cover or blanket over the heel of a sloped base lessens the chance of rupturing the cover as the wall moves under load.

(3) The resultant of applied forces is more nearly normal to a sloped base, thereby reducing the tendency of the structure to slide along that plane.

(4) A full-size flood wall test performed by the Ohio River Division (1948-1956) (U. S. Army Engineer Division, Ohio River 1958) indicated that the sloped-base wall moved consistently less than the horizontal-base wall of comparable design.

c. Selection. Both base types have their advantages and disadvantages. Final selection will depend upon the specific site conditions at the project under consideration.

7-6. Horizontal Water and Earth Loads on Keys. For flood walls on clay foundations, full flood head will be conservatively assumed to act at the bottom of the key and the horizontal water load acting on the riverside face of the key will be computed on this basis. The seepage path will then be assumed to begin at the bottom of the key. The landside face of the key will normally be assumed to be in full contact with the earth-resisting movement of the wall.

7-7. Unsuitable Foundation Material and Bank Stability. Foundation material found to be unsuitable may be avoided by a change in alignment or may be removed and replaced with suitable earth fill (Figure 7-5). The wall may also

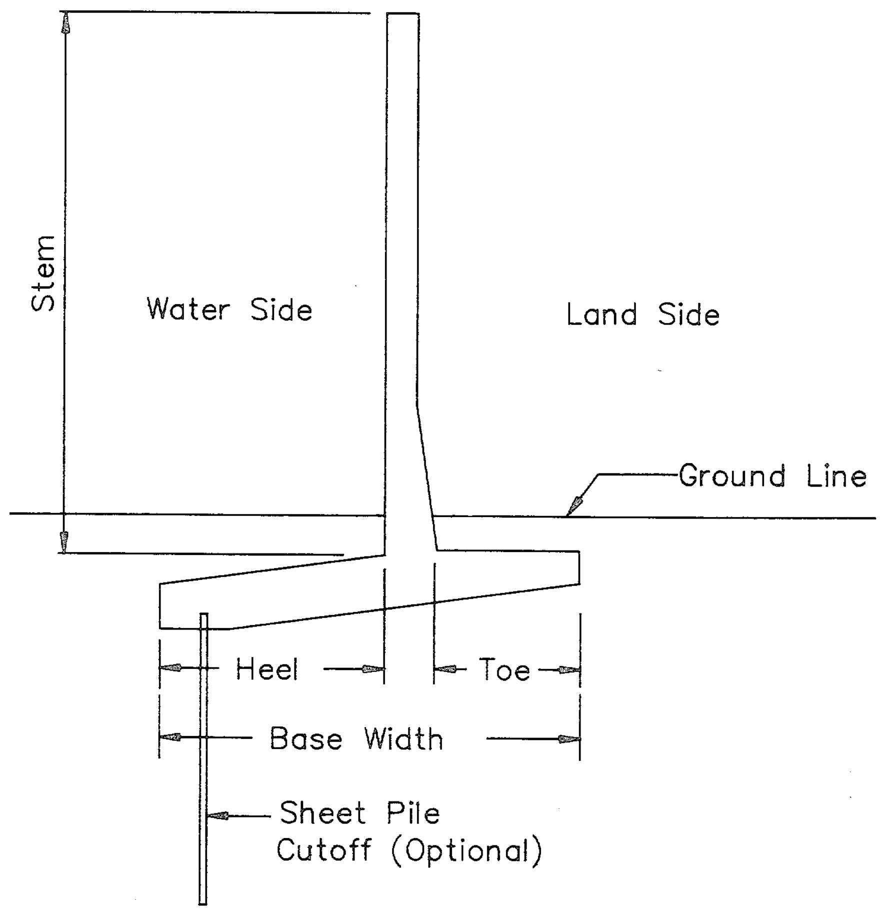



Figure 7-4. T-type flood wall--sloped base

be founded on piles through the unsuitable material. In some cases the removal of unsuitable foundation material involves the removal of or cutting into the existing riverbank on which the flood wall is to be placed. In other cases the right-of-way may be so restricted and confining that the flood wall may have to be placed near the top edge of the bank or even riverward of the bank. In those cases, fill placed riverward of the top bank is permitted, if proper precautionary measures are taken. Careful attention must be paid to the outlining of and removal of unsatisfactory material and to the selection of suitable replacement material. New material must be obtained, placed, and compacted to provide adequate support for the flood wall. Replacement material should undergo the same types of laboratory testing as existing foundation material. Placement and compaction techniques should generally be in accordance with earth dam and levee requirements. Slopes steeper than 1.0V on 1.5H and areas that require hand compaction should be minimized. Slopes on which there is evidence of past instability, or in which fill is a component,

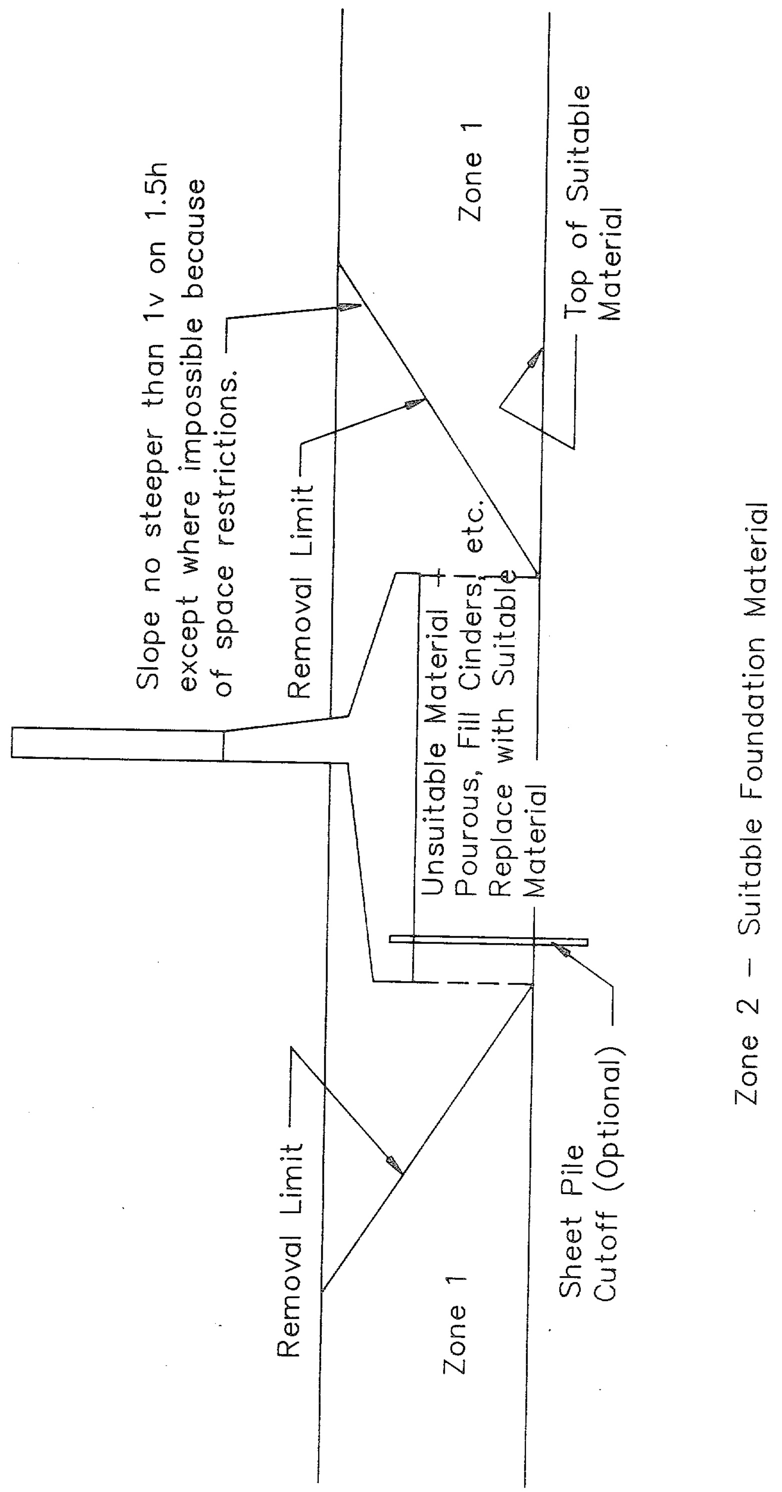



Figure 7-5. Removal limits of unsuitable foundation material

should be investigated for stability. All riverward slopes should be checked for stability if the failure of the bank would jeopardize the stability of the wall.

7-8. Scour Protection. Occasionally a flood wall is exposed to scouring because of the direction, curvature, and velocity of current or waves, characteristics of the soil, topography, etc. Scouring at the wall footing should be considered, and where anticipated, protected with riprap. Design guidance on sizing riprap is given in EM 1110-2-1601.

## Section IV. Types of Monoliths

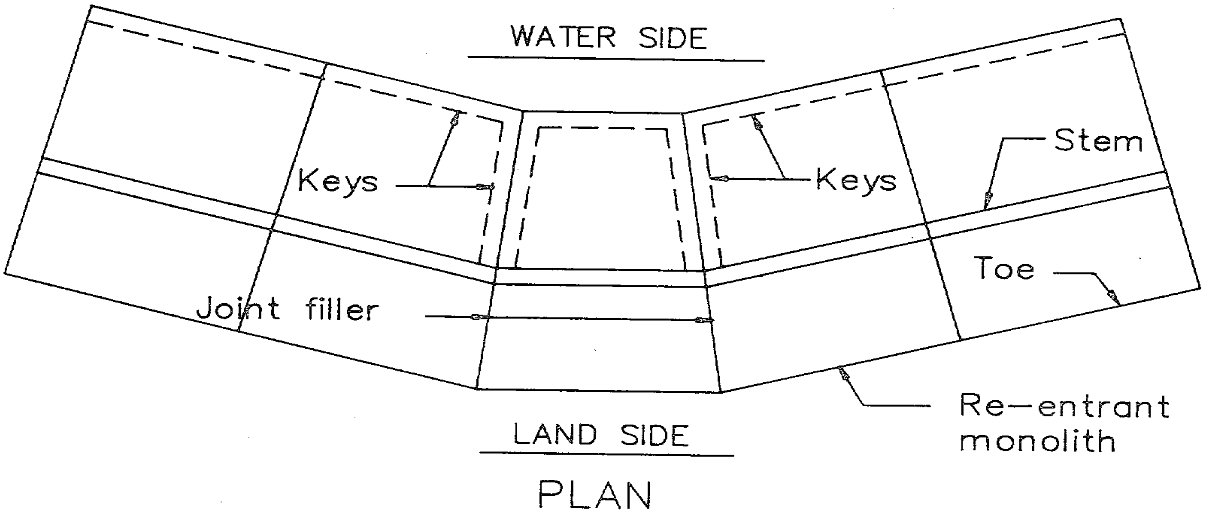
7-9. Change-of-Alignment Monoliths. Changes in alignment require special monoliths (Figure 7-6). Monoliths with less than a 10-degree change (horizontal) do not need to be analyzed as a special category. Monoliths of short length or abrupt alignment changes may require very wide bases. A 90-degree corner monolith is an indeterminate structure. Adjacent monoliths should not be considered to provide resistance in the stability analysis.

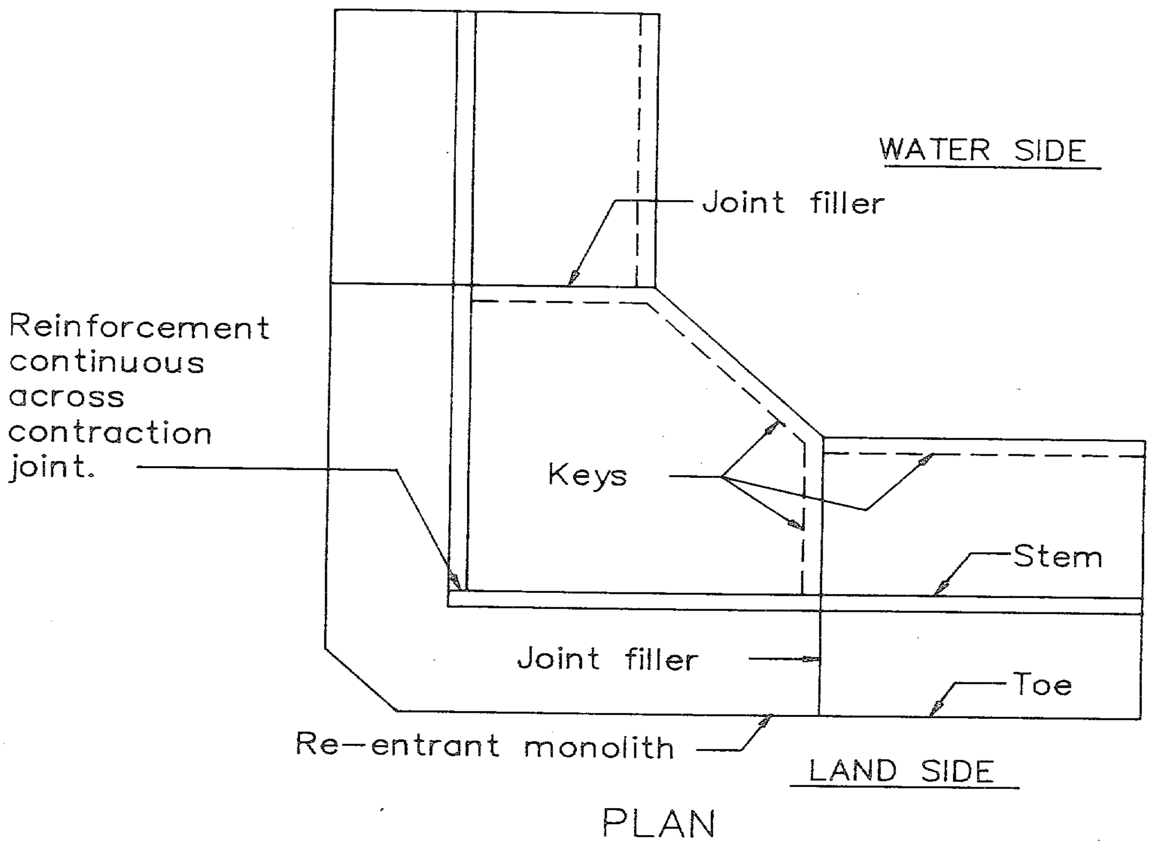
7-10. Closure and Abutment Monoliths. A number of openings must be provided in many flood walls. The openings provide access for commerce, safety, and recreation during periods of low river stages. The number and size of openings depend on local requirements. Each opening must be provided with a moveable closure structure. During flood periods, the closure structure is installed on base and abutment monoliths (this combination is a special monolith). These special monoliths must be designed both for the design water load at high water and traffic loads during low-water periods.

7-11. Drainage Structure Monoliths. When topography, foundation conditions, and economics permit, it is preferable that structures housing gates and pumps be designed as integral parts of the flood wall. These special monoliths must be designed to minimize differential settlement across a monolith or between adjacent monoliths. For closure gate requirements and the need for secondary closure gates for drainage outlets, see EM 1110-2-1410.

7-12. Transition Sections Between Flood Walls and Levees.

a. Junctures. A junction between a T-wall and levee is not made directly or abruptly, but with a short transition concrete-capped sheet piling I-wall between the two (Figures 7-7 and 7-8). One of the primary concepts in the development of this transition is to arrange details so there will be a minimum amount of differential movement of joints of monoliths in the transition. The levee end of the transition will usually settle a considerable amount, due primarily to foundation consolidation under the added weight of the levee. The T-wall monolith immediately adjacent to the beginning of the levee adds far less superimposed weight on the foundation. Hence, there is much less settlement at this end of the transition. The I-wall can be satisfactorily adopted as a transition section between levee and T-wall because this type of construction can, and in fact must, be done after completion of the levee. A delay in inserting the I-wall allows for settling of the levee,




Expansion joint material (paragraph 7–14a) to be used where each end of re–entrant monolith touches the adjacent monoliths (see Figure 7–9a).

Figure 7-6. Return keys on reentrant monolith

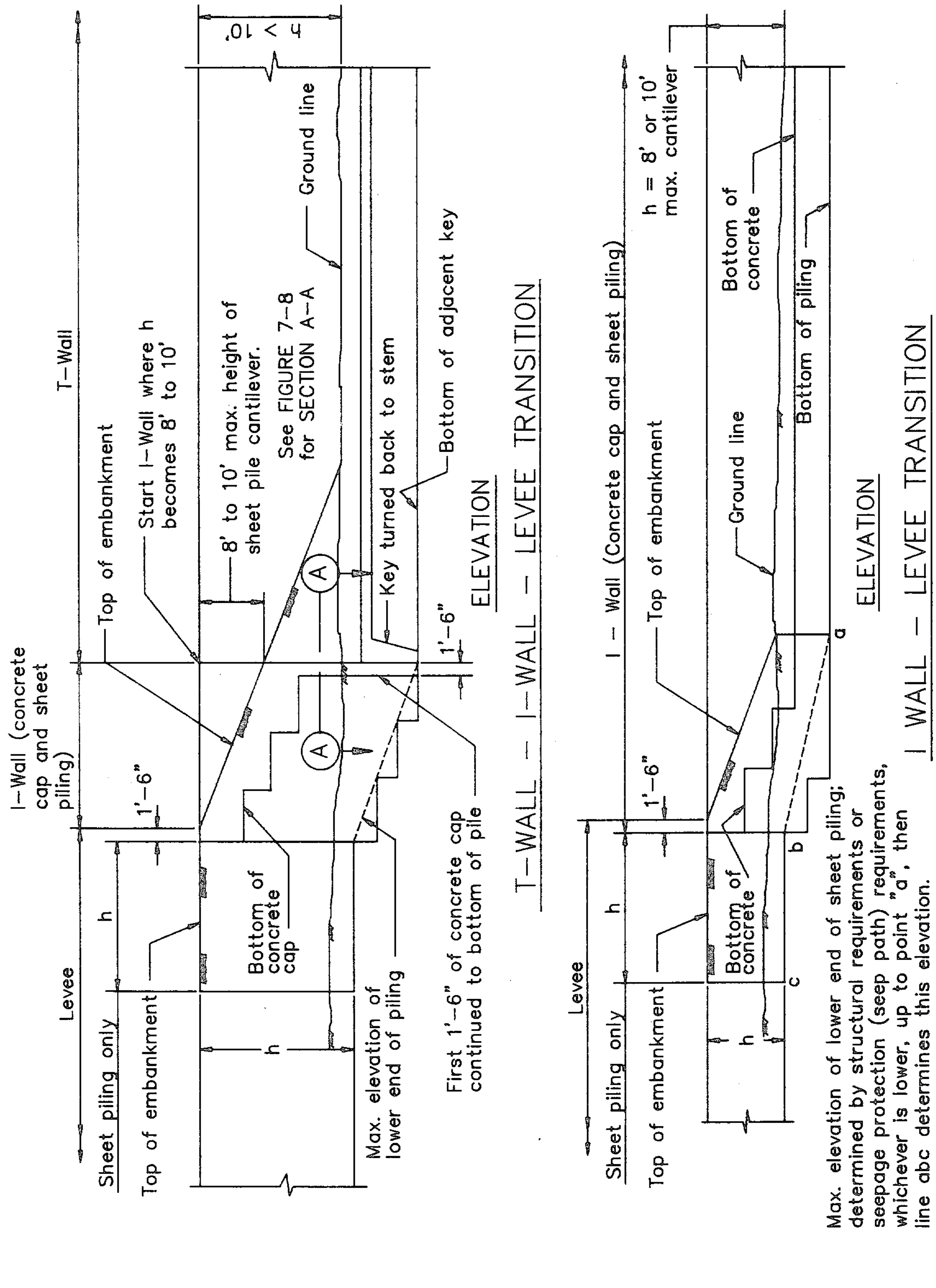
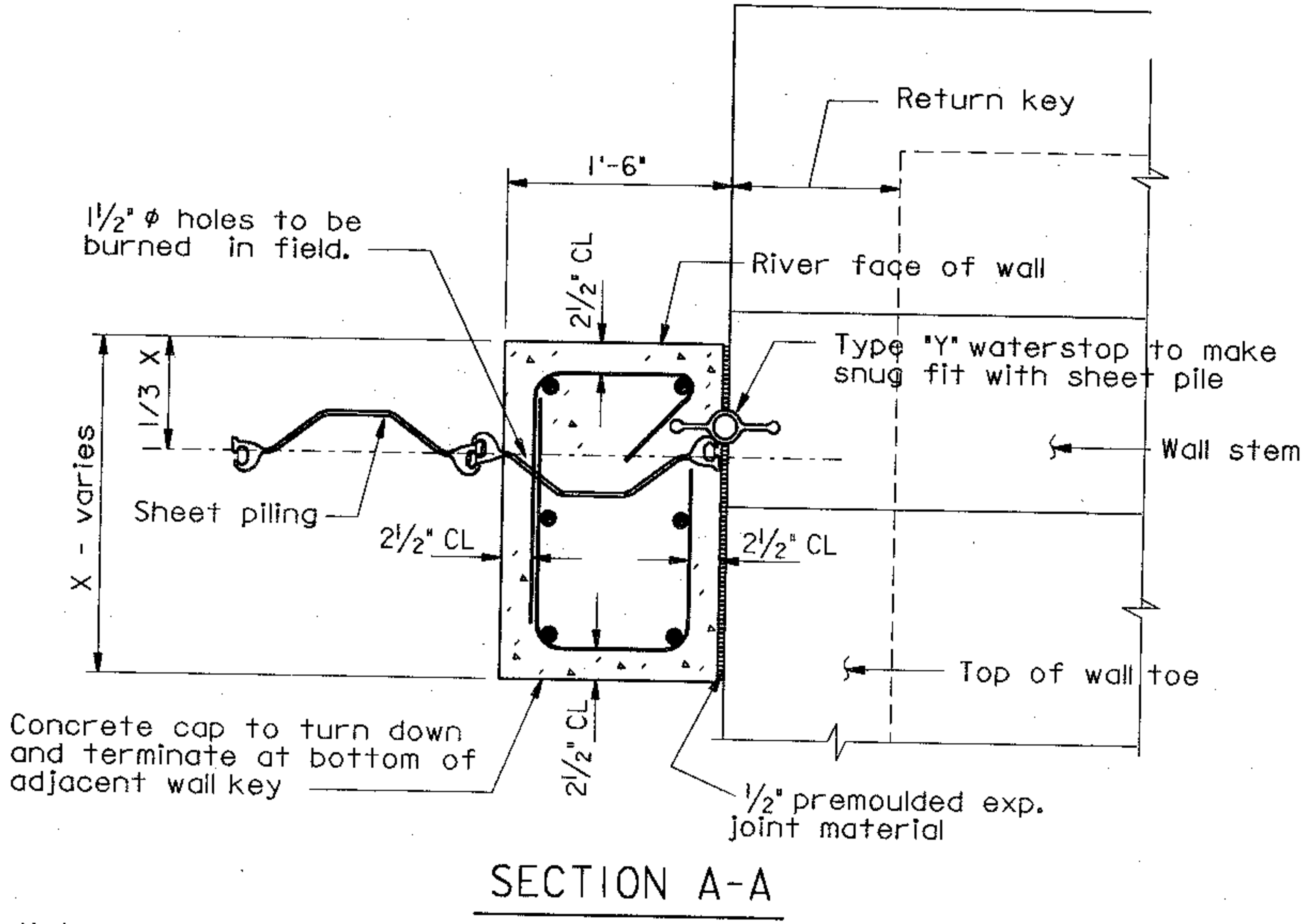



Figure 7-7. Flood wall-levee transitions




Note:
For location of section A-A, see Figure 7-7.

Figure 7-8. Typical detail of joint between I-wall and T-wall thus lessening the differential settlement between the levee end of the transition and the T-wall. Review by hydraulic engineers is required for inland flood walls to assure that the transition geometry will not create significant flow disturbance with consequent scour.

b. I-wall. The I-wall portion of the transition is begun where the levee slope (parallel to the protection) reaches a point 10 feet below the top of the wall. In cases where protection is already 10 feet or less above the levee, an I-wall, if used, is merely continued into the levee as shown in Figure 7-7.

c. Sheet Piling. It should be noted in Figure 7-7 that the sheet piling is continued into the levee for a specified distance beyond the last concrete cap.

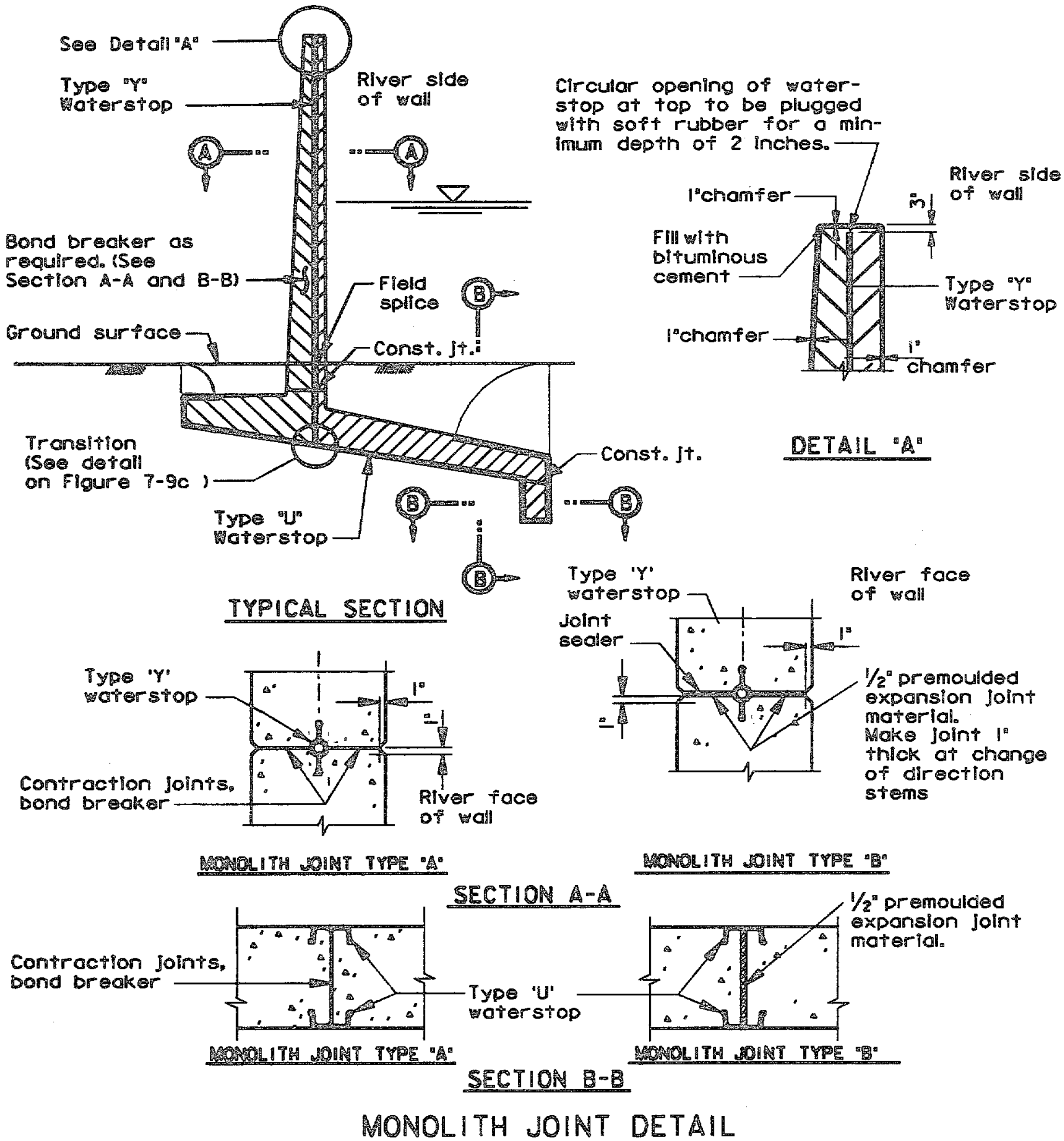
## Section V. Water Stops and Joints

7-13. Water Stops. As shown in Figure 7-9a, b, and c, for yielding foundations a U-shaped (type "U") water stop should enclose almost the entire base and a center bulb (type "Y") water stop, located in the stem, is joined to the U-shaped water stop at the bottom of the stem. Experience has shown that a center bulb or dumbbell water stop located within the base section is likely to allow excessive seepage. Between monoliths on a foundation requiring a cutoff, the type Y water stop in the stem should be extended to tie into the cutoff, and the type U water stop around the base should be deleted. The earth surface on which a type U water stop is installed must be firm and smooth, with no chips, sags, humps, clods, or loose debris that would prevent intimate contact between the water stop and soil. See Chapter 6, paragraph 6-4e, for general guidance on water stops. Because field construction problems are common for the type "A" joints shown in Figure 7-9a with the type U water stop shown in Figure 7-9b, and because the buried base slab does not experience wide temperature changes, an optional base slab joint is allowed when the base is placed. This base slab joint uses construction joints without water stops but with the base slab longitudinal reinforcement continuous through the joint. When this option is used, longitudinal reinforcement of at least 0.4 percent of the slab cross-sectional area must be provided in the base slab, half in each face, but with not more than #9 reinforcing bars at 12-inch spacings in each face.

7-14. Contraction and Expansion Joints. Contraction and expansion joint details are illustrated in Figures 7-9a through 7-9c. Contraction joints (type A) should contain a bond-breaker. Expansion joints (type "B") should contain 1/2-inch preformed expansion joint filler in:

a. All protruding (convex on water side) monolith bases, and in selected reentrant monolith bases and stems as shown in Figure 7-6.

b. In bases and stems of alternate monolith joints in straight-line runs, if warranted by previous experience with similar foundation conditions.

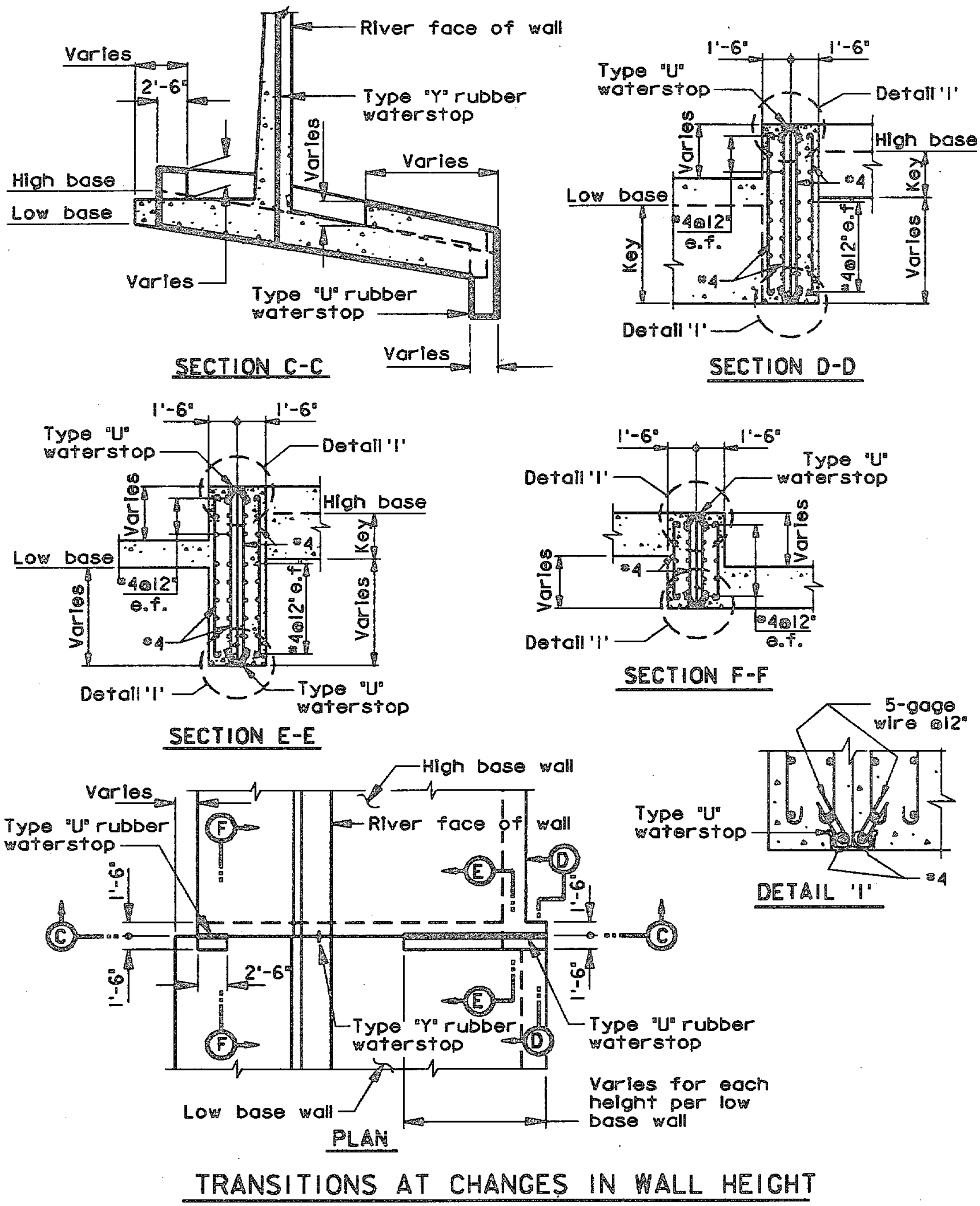



NOTES:

1. Extreme care should be exercised in placing type "U" rubber waterstop to insure firm contact with the prepared subgrade throughout its entire contact area.
2. Type "A" joint used in straight runs of wall, 30 feet spacing
3. Type "B" joint used in junctures of wall with gate wells, pump stations and gate abutments, and in change of direction monoliths.

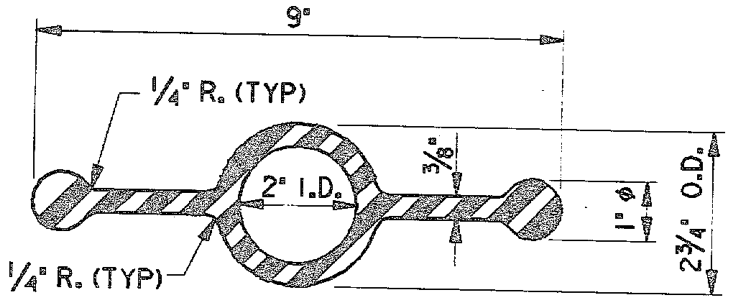
a. Monolith joint detail

Figure 7-9. Typical joint and water stop details (Sheet 1 of 3)

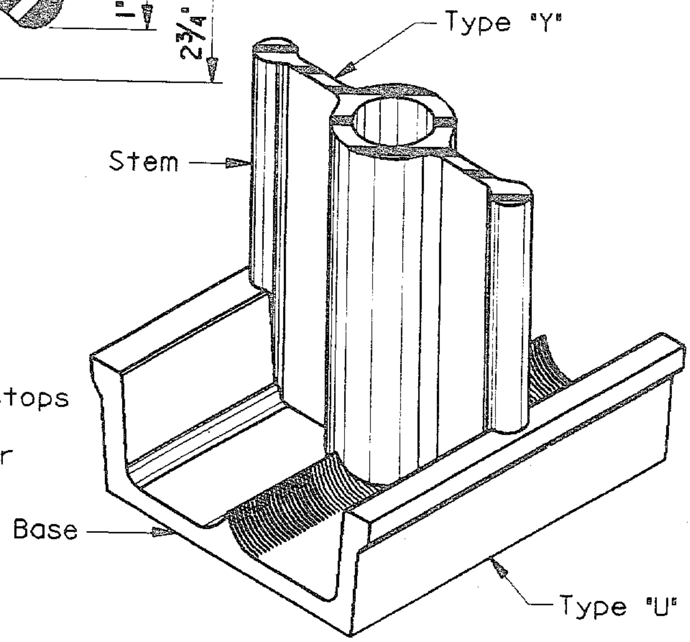



b. Transitions at changes in wall height

Figure 7-9. (Sheet 2 of 3)

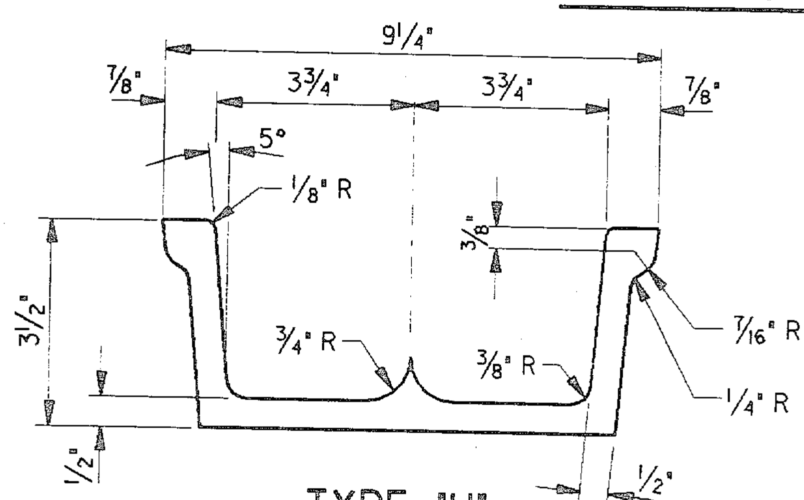



TYPE "Y"




The Type "Y" and Type "U" waterstops shall be joined by vulcanizing if rubber waterstops are used ; or by heat sealing the joint if PVC waterstops are used.

TRANSITION BETWEEN STEM AND BASE
ISOMETRIC SKETCH




TYPE "U"

c. Water stop details

Figure 7-9. (Sheet 3 of 3)

c. In bases and stems of junctures of walls with gate wells, pump stations, gate abutments, and similar structures. Nonflexible material in a protruding angle joint is particularly dangerous.

d. See paragraph 6-4 for general guidance on joints.

## Section VI. Site Considerations

7-15. Adjacent Structures and Rights-of-Way. Flood walls are usually built because only a narrow right-of-way is available. The presence of existing buildings or other structures is usually the reason for a narrow right-of-way. Sewer pipes with open joints, structures with basements, and excavations close to the wall may create a hazard to the safety of a flood wall. Also, new structures that are built close to existing flood walls can create the same hazards. Present right-of-way acquisition policies do not permit legal restrictions to be placed on future construction; however, local interests should be advised in writing of potential hazards, of required design and construction measures, and should be requested to closely supervise new construction close to the flood wall. Potential hazards can be avoided by proper design and construction measures. One hazard that should be considered is seepage. A basement or other excavation on the landside of the flood wall may result in shortened seepage paths. A basement or excavation on the riverside may also create a safety hazard if it penetrates the impervious blanket or shortens the seepage path. When feasible, the basement or excavation should be backfilled with the same type of material existing in the foundation of the flood wall. If relief wells are selected to control seepage they should be located, if at all possible, between the flood wall toe and the adjacent structure. Protection of the basement area may require lowering of discharge elevations for safeguarding the wall. The location of relief wells within a basement area is not prohibited, but it leads to problems of construction, maintenance, and discharge collection. If the seepage problem is only one of quantity, sump pumping may be used during periods of high water. A second hazard that landside basements and excavations create is to lessen the resistance to sliding along a foundation failure plane. For this reason potential planes of sliding into basements or excavations should be studied. If backfilling is not possible, other measures include the addition of fill between the stem and the building or strengthening the basement to provide the needed resistance. Riverside excavations which contribute to riverward foundation instability should be backfilled, at least to the extent that stability requirements will be satisfied. For the special situation where a wall in a congested location is subjected to an unusually large horizontal force, such as the force of a breaking wave, T-type flood walls are frequently worth the extra cost over other types of construction. This situation requires an unusually wide base for sliding stability, requiring more right-of-way and, hence, more cost for construction. The relatively thin stem of the T-wall does, however, provide the most usable surface area adjacent to the stem after backfilling, in comparison with embankment, braced walls, etc., making the T-wall the preferred solution in spite of the extra construction easement right-of-way. While an I-wall also provides little intrusion on the completed surface area, its use can be precluded by the pile-driving vibration and consequent chance of damage to adjacent structures.

7-16. Architectural and Landscaping Considerations. Aesthetics should be considered in the design of flood walls, from the standpoint of blending the project with the surroundings. Whenever possible, the wall should appear to be a natural extension of the local topography. The basic design of these structures should be a coordinated effort between the design engineer, the architect, and the landscape architect. While it is seldom feasible to preserve the natural setting intact, design techniques and careful construction methods can be used to protect or even enhance the aesthetic value of the immediate project area. Landscape planting design for project structures should consider the entire area affected by the contemplated construction. Further details may be found in EM 1110-1-2009 and EM 1110-2-301.

## Section VII. Instrumentation

7-17. General and Specific Considerations. Flood wall instrumentation should be considered so that performance can be monitored, particularly during periods of high water. The decision on how much, if any, instrumentation is appropriate must be based on these factors:

a. Who will monitor and evaluate the instruments--Corps of Engineers, local interests, etc., and how meaningful their evaluations are expected to be.

b. Access to the instruments during flood conditions, especially during hurricane-flood situations where high winds may make accessibility impossible.

c. Time required for meaningful evaluation, compared with the expected duration of the flooding.

d. Other particular considerations for specific situations.

The instrumentation descriptions that follow must be implemented in light of the above decision. Specifically, areas with high walls, low embedment ratios, replaced foundation materials, overbank fills, pervious materials in the foundation, and changes in direction should be considered for instrumentation. When founding a flood wall on earth, the distance between monoliths with piezometers should not exceed 1,000 feet unless warranted by site conditions. Properly installed, maintained, and observed instrumentation can forewarn of dangerous conditions that may affect the stability of the structure. All instruments should be read soon after construction is complete. Knowing the as-built conditions of the wall is essential for an accurate determination of later behavior. Initial piezometer readings should be repeated until equalization (steady state) occurs. All instrumentation readings should be made by trained survey or flood-patrol personnel. Ideally, all instruments will be read frequently during high water stages. During design floods, the procedure may prove almost impossible because of the need for trained personnel to direct flood fights; but readings should be made at certain, previously selected, critical locations during design flood stages. During normal water stages, instruments should be read prior to district periodic inspections so that the inspection party has the necessary evaluation data. Such data also provide a history of flood wall reactions over the years, during both high and normal water. Information concerning frequency and manner of conducting periodic inspections and evaluations is contained in ER 1110-2-100, while ER 1130-2-339 covers local flood protection projects.

7-18. Types of Instrumentation. The principal types of flood wall instrumentation monitor movements, both vertical and horizontal, and hydrostatic pressures in the foundation. The instruments selected should be simple to install and observe, and efficient in performance and functional reliability. The monitoring of the movements provides an indication of possible sliding instability or possible water stop rupture. The piezometers provide a record of hydrostatic pressures in the foundation which can indicate uplift and possible excessive seepage pressures. Instrumentation systems, installations, and devices are discussed in detail in EM 1110-2-4300.

a. Movement Monitoring. All reference points to monitor movements should be tied in to a permanent baseline located so that it is unaffected by movements of the wall. When establishment of a baseline is not feasible, the relative movements observed between monoliths or by means of triangulation can provide valuable data on behavior of the wall. Reference points to monitor the wall movements need to be installed during construction. Noncorrosive metal plugs should be installed in the top surfaces of the stems within 6 inches of each end of each monolith. The reference marks in the plugs of four to six successive monoliths should be placed in a straight line with theodolite or stretched wire. At changes in alignment, the straight line should be continued until it intersects the far side of the next monolith and a reference point for alignment control is placed. Each plug's changes in horizontal movement and elevation should be measured to 0.001 foot. Stations to be read with electronic optical reading devices need to be established at locations near the ground surface level on the landside of the stem. Selection of electronic-optical station locations for the stem should be based on factors such as changes of direction, areas of overbank fill, foundation replacement, high walls, low embedment ratios, and junctures of flood walls with drainage structures. The monitoring system selected should be vandalproof. In many cases the monitoring system can be tied into the same baselines established for the reference markers on top of the wall. Tilting of stems can be measured by a tiltmeter.

b. Foundation Piezometers. Design, installation, and observations of piezometers are described in EM 1110-2-1908, Part I. The simplest, most reliable method of measuring pore water pressures is the open tube piezometer. For impervious soils, the Casagrande type of piezometer with 24-inch-long porous stone is recommended. In order to measure the piezometric pressure at the porous tip, the boring for installation of the Casagrande piezometer must be effectively sealed against migration of seepage along the piezometer riser. For semipervious to pervious soils, a driven wellpoint type of piezometer is recommended. Where possible, the wellpoint should be driven into undersized, pre-bored holes. More piezometers can be added if foundation conditions warrant.

## Section VIII. Operation and Maintenance Manual Requirements

7-19. General Coverage. General coverage of the requirements of local cooperation is contained in EM 1120-2-109. As written, the regulations are general in nature and obviously cannot give detailed instructions for the maintenance and operation of a specific project. Therefore, it is necessary for the district office having jurisdiction over the specific project to issue an adequate operation and maintenance manual for the guidance of local interests.

## Section IX. Review of Existing Flood Walls

7-20. Inspection. Flood walls should be examined during scheduled periodic inspections, after major periods of high water, and when special events warrant an inspection (building or excavating near the wall, etc.). A determination of areas which may be weak or critical from the standpoint of leakage and stability should be made. Criteria for this determination are described below. Areas deficient in any of the criteria will be considered weak or critical, depending on the degree of deficiency.

a. Horizontal Movement. Areas in which movement of a straight section of monoliths or differential movement between any two monoliths is greater than expected will be considered critical.

b. Joint Opening or Spreading. Joints referred to in this paragraph are those having a water stop embedded in the interior of the section. Using the results of the full-size flood wall test performed by the Ohio River Division, (ORD) in 1955, expected spreading of joints at 90-degree reentrant corner monoliths (concave on the riverside) will be 42 percent of the expected movement of the straight run walls. Not only may joints at corner monoliths become critical upon application of load, but open joints below ground should be considered critical. Any joint can become open through loss of joint filler or through unequal settlement between adjacent monoliths or structures such as levees, pump houses, gate wells, and gate abutments. Some joints below ground may need to be excavated to determine the adequacy of joint filler. If the expected joint opening is greater than the allowable, the area should be considered critical.

c. Foreign Material in Joints. The presence of inflexible foreign material, such as grout and pieces of aggregate, in expansion joints is dangerous from two standpoints. Grout, particularly if located within the fold of the water stop, destroys the flexibility of the water stop and, upon the occurrence of differential movements, allows the water stop to be torn. Grout and pieces of aggregate anywhere in the joint prevent the joint from fulfilling its expansion function. This condition becomes particularly dangerous at protruding angle locations; i.e., where the wall appears convex when viewed from the river. Here, the wall may be tilted waterward by a wedging action upon expansion of adjacent monoliths in hot weather. This wedging of adjacent monoliths at changes in alignment is likely to force excessive flexure in the stem, sufficient to cause failure. The same tilting can occur at reentrant monoliths (Figure 7-6), but there the tilting is landward and the reinforcing is more adequate to resist the stress. For angle monoliths protruding toward the river, the landside temperature steel can be quickly overstressed.

d. Water Stops. Joints with torn or parted water stops should be considered critical. Torn water stops may not be noticed during an inspection, particularly if the joint has not spread open. If sufficient differential movement has occurred, it should be assumed that the water stop is torn. The amount of tearing to be allowed should be based on factors causing piping; however, this is very difficult to predict. In the above cases, if a total differential movement (transverse and longitudinal combined) of 1/2 inch or more has occurred, the water stop should be considered torn unless shown otherwise.

e. Foundation Voids. All unequal settlements should be viewed with suspicion. In particular, unequal settlements adjacent to structures such as pump houses and gate wells should be the subject of rigid examination. Usually one or two monoliths (or a portion of one monolith) are constructed on compacted fill in these areas. Initial unequal settlement may cause the first monolith to bridge or wedge between the second monolith and the other structure. Further consolidation of the fill then leaves a dangerous void or voids under this base. Only underground examination will reveal the presence of these voids.

f. Stability Analyses. Original seepage assumptions or patterns should be reviewed for realistic representation of actual foundation conditions. Particular attention should be paid to foundations having pervious strata which connect directly with the river. Where indicated, seepage and/or stability analyses should be recomputed as described in Chapters 3, 4, and 5. In addition to a recomputation of uplift, the shear strengths used in the original analyses should be reevaluated on the basis of a study of types of soil and their drainage and consolidation characteristics. In cases where there is a lack of sufficient foundation information in areas suspected to be weak, new soil samples should be obtained as close to the existing wall as is feasible. Areas found to have questionable stability should be closely observed during high floods.

g. Basements and Other Excavations. The seepage aspects and the foundation stability of walls which have had basements excavated on either side of and adjacent to the wall since the original design and construction were completed should be investigated.

h. Seepage Conditions Landside of Flood Walls. These areas should be investigated thoroughly and seepage control of pressure relief provided, if needed.

7-21. Repair Measures.

a. General. The following repair measures are only suggestions. Their use is not mandatory if more feasible or economical measures can be devised for the individual problems involved.

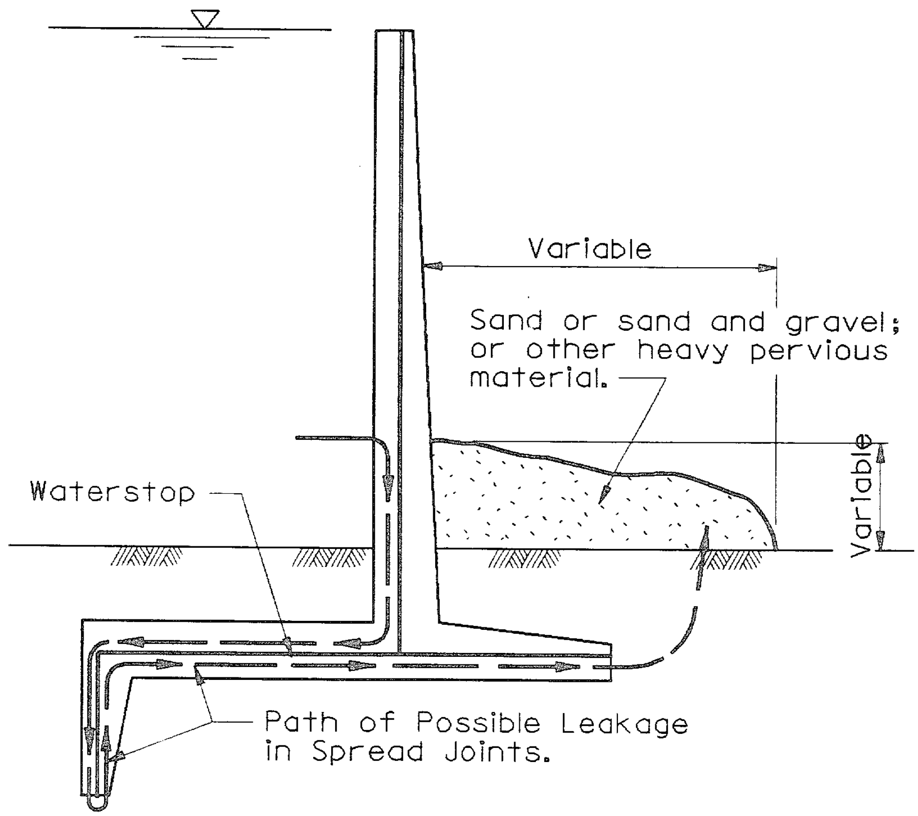
b. Additional Landside Cover. The most obvious and straightforward method for reducing anticipated horizontal movement or increasing sliding stability is the addition of landside cover or fill to the wall (see Figure 7-10). At locations where additional landside fill is not feasible or possible due to highways, railroads, and other structures, measures to reduce seepage pressure (such as those described below in paragraph 7-21d) will have to be employed to decrease landward movement or increase sliding stability.

c. Additional Waterside Cover. In areas where earth cover over the waterward end of the heel is deficient, the recommended remedy is the addition of cover.

d. Supplemental Water Stops. The supplemental water stop scheme shown in Figure 7-11, a and b is a means of correcting for torn water stops, open joints, and possible earth cracking over the key because of thin heel cover or excessive movements. The sheet piling shown in the scheme is necessary to provide additional cutoff to compensate for loss of part or all of the normal seep path between earth and the waterside face of the key. The pile cap should be placed at the bottom of the key to limit excessive leakage of water around the upstream and downstream ends of the pile curtain as the wall moves landward under load. Another possible method of repair is to seal the opening below the existing water stop in the base by injecting cement grout. The opening above the water stop in the base could be sealed with an elastic sealant such as polysulphide elastomer.

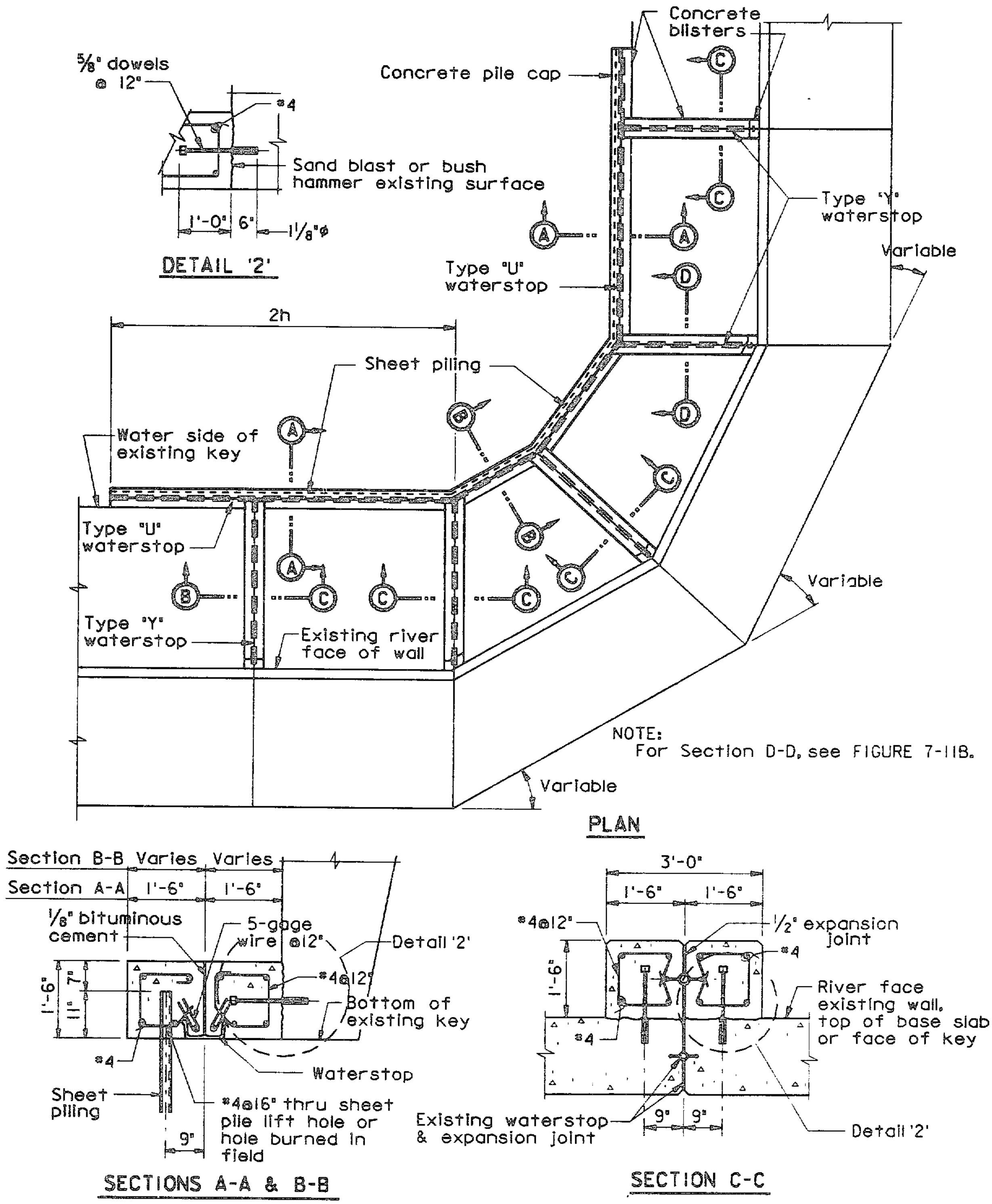
e. Other Problem Areas. Foreign incompressible material in the joints should be removed by the most expedient method. Riverside excavations near the heel should be backfilled with impervious material if it is suspected that dangerous seepage conditions may occur during high water.

f. Overtopping Scour Control. For coastal walls or other walls where scour has removed landside cover, consideration should be given to placing concrete slabs over the restored cover within a distance of 20 feet from the wall stem.



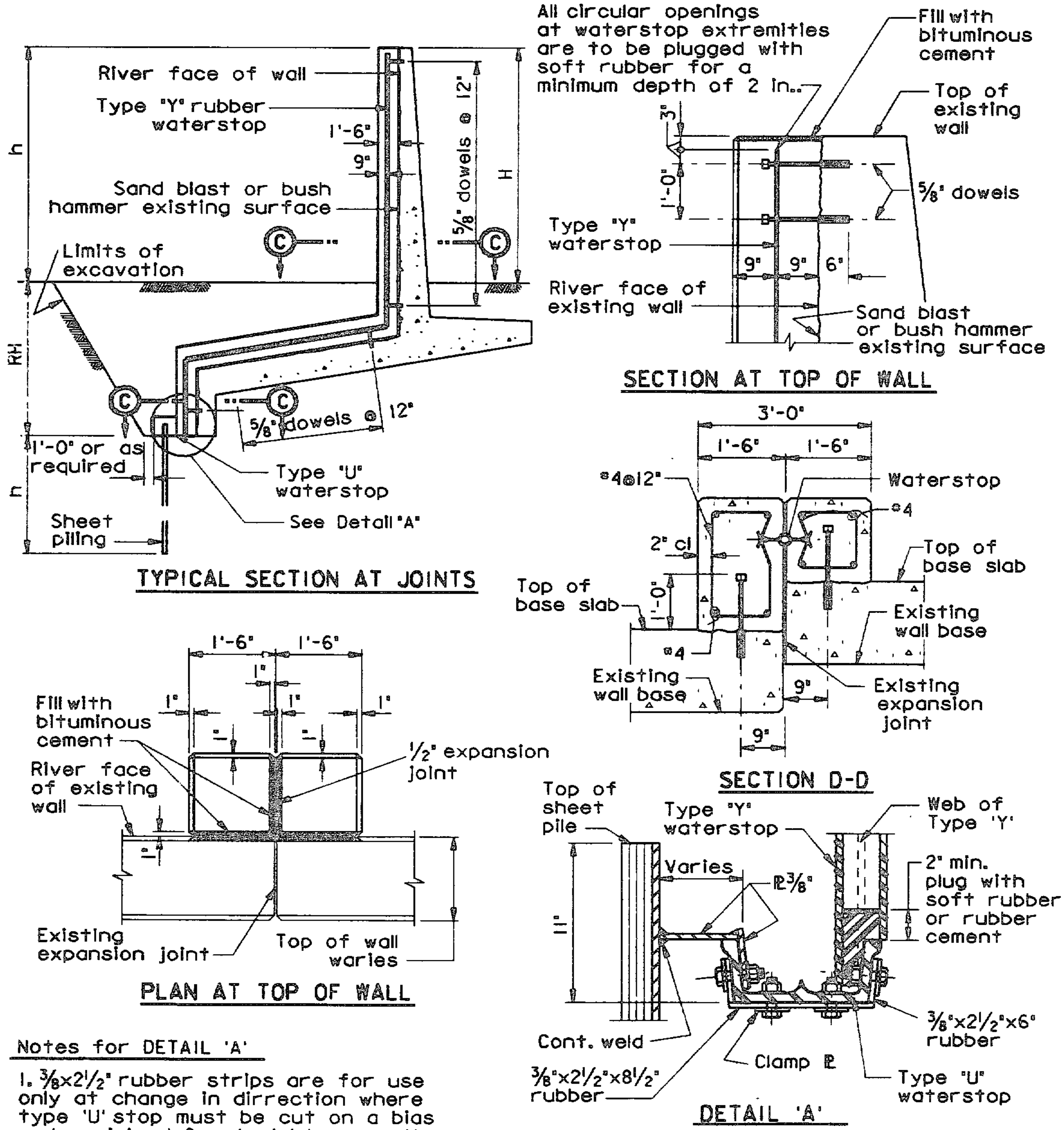

Note: The area and depth of material shall be sufficient to prevent loss of foundation material as determined by observation of the outflow.

Figure 7-10. Emergency measures to control piping




a.

Figure 7-11. Permanent water stop repair measures (Continued)




Notes for DETAIL 'A'

1. 3/8x2 1/2" rubber strips are for use only at change in dirrection where type 'U' stop must be cut on a bias and re-joined. On straight runs these strips need not be used.
2. Bulbs of type 'Y' stop are shaved down to web to provide a flat surface for bolting to inside of type 'U' stop.
3. All rubber surfaces in contact with each other are coated with rubber cement.

NOTES:
1. For details of "Y" & "U" type rubber waterstop, see FIGURE 7-9C
2. All steel in blisters 2" clear.
3. For Section C-C, see FIGURE 7-11A
4. For Section D-D location, see FIGURE 7-11A

b.

Figure 7-11. (Concluded)